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16. Abstract

The accuracy of saturation flow values is of prime importance when determining the capacity of signalized intersections. The objective of this study was to collect a large sample of field measurements so that reliable saturation flow values could be obtained and the factors affecting saturation flow could be identified.

The results identified several factors which had significant influence on saturation flow. A formula was recommended for use in estimating an appropriate saturation flow value for a specific lane on an approach to an intersection. Adjustment factors were applied to a "base" saturation flow value. Adjustments were developed for the following factors; location in city, city population, vehicle type and turning maneuver, gradient, width of lane, turning radius (for right-turning vehicles), pedestrian activity, type of lane, speed limit, and light condition.

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ANALYSIS OF SATURATION FLOW AT SIGNALIZED INTERSECTIONS

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INTRODUCTION

When the green signal phase begins on an approach to an intersection, vehicles take some time to reach a normal running speed; but after a few seconds, the queue discharges at a more or less constant rate termed saturation flow. The basic model of the variation of discharge rate of a queue with time in a fully saturated green period is illustrated in Figure 1 (1). A fully saturated green period is one in which the queue is not completely discharged during the green period. Saturation flow is the maximum constant departure rate from the queue during the green period, and it remains fairly constant until either the queue is exhausted or the green period ends. Saturation flow may vary as a function of items such as layout of the intersection (lane width, grade, etc.), number of turning vehicles, and types of vehicles in the traffic stream.

Accuracy of saturation flow values is of prime importance when determining the capacity of signalized intersections. The objective of this study was to collect a large sample of field data so that reliable saturation flow values could be computed and factors affecting saturation flow could be identified. These saturation flow values may be used as input for determining intersection capacity and when using computer models to simulate and optimize signal systems.

REVIEW OF LITERATURE

An extensive literature review was conducted on the topics of intersection capacity and saturation flow. This review identified various methods which had been used to measure saturation flow. It also identified many factors which had been found to affect saturation flow. These factors were:

- vehicle position in queue,
- location in city,
- city size,
- vehicle type and turning maneuver,
- gradient,
- lane width,
- turning radius,
- pedestrian activity,
- parking,
- approach width,
- type of lane,
- time of day,
- speed limit,
- weather or road surface condition, and
- light conditions.

A detailed summary of results of the literature review is in Appendix A.

PROCEDURE

DATA COLLECTION

Data collection consisted primarily of measuring time intervals between when the signal turned green and when the rear wheels of each vehicle in the queue crossed the stop bar. The stop bar was selected as the screenline because it was felt that it would give the best and most consistent results. At many intersections, cross-streets were offset or intersected at an angle, which made determination of point of vehicle entry into the intersection difficult. The major problem encountered with using the stop bar occurred whenever a vehicle stopped past that line. In such a situation, the time for that vehicle was omitted, although its presence was noted.

The first phase of the study involved selecting intersection approaches where data would be collected. The majority of data was collected in Lexington, Kentucky. Approaches were selected so that a range in values for the variables would be available for data analysis. In other words, an attempt was made to select approaches in different areas having a range in such variables as lane width and gradient. Various cities across the state were selected for data collection. The objective was to select cities which had a wide range of populations and were distributed throughout the state. Data describing each approach were recorded onto a data sheet (Figure 2). All necessary measurements were made at subject approaches. Approach grade was obtained using an Abney hand level meter.

The data sheet used for collecting actual saturation flow data is shown in Figure 3, and descriptions of codes used thereon are given in Figure 4. The data sheet contained information describing the intersection, in addition to basic information obtained for each vehicle in the queue. For each vehicle, the time from start of green to when the rear wheels crossed the stop bar was recorded (t). A description of the vehicle and/or its action was recorded when appropriate (d). Specifically, when a vehicle turned, when it was anything other than a passenger car (pick-up trucks and vans were included with cars), or when its progress was interrupted, the appropriate code, as described in Figure 4, was noted. Vehicles that were interrupted were excluded from the subsequent analysis.

Data were collected for each vehicle in the queue and recorded as a function of the vehicle's position in the queue (n). When vehicles changed lanes or entered the queue from an adjacent driveway, thereby disrupting normal movement, data collection was discontinued for that cycle. When more than 25 vehicles went through the intersection during a signal phase, the time and number of the last vehicle were recorded. Data were collected only for those vehicles that were part of the queue when the signal indication turned green or became a part of the queue before reaching the stop bar.

If the entire green phase was utilized without exhausting the queue, the phase was termed "loaded", and that was noted. Green-plus-yellow times were noted for loaded cycles to determine the lost times at the ends of cycles. This will be used in intersection capacity calculations. In many instances, the green-plus-yellow time was constant for loaded cycles. The yellow time was also noted for most approaches.

All times for individual vehicles were obtained with a split/cumulative timer that displayed time to the nearest 0.01 second. The timer had a digital display which was easy to read. The timer was started at the beginning of green and a button was pushed when each vehicle crossed the stop bar. Elapsed time since starting of the timer was displayed for each vehicle and noted to the nearest 0.1 second on the data sheet.

DATA ANALYSIS

Data were transferred from data sheets to a computer file by entering at a remote terminal. The format used is shown in Figure 5. Explanations of various codes are given in Figure 6. All data for a particular lane were grouped and preceded by a header card that contained information concerning that intersection, approach, and lane. Inapplicable fields were left blank. The field for "headway" on the data records was not entered initially; a computer program was used to calculate headways and enter them in that field. The headway for a specific vehicle was calculated by taking the time recorded when that vehicle's rear wheels crossed the stop bar and subtracting the corresponding time for the preceding vehicle. For example, the headway for the tenth vehicle would be the time between the ninth and tenth vehicles. The headway for the first vehicle was the time between the onset of green and the rear wheels of that vehicle crossing the stop bar. Headways tend to be highest for the first vehicle in the queue, and decrease for succeeding vehicles until reaching a constant level. Saturation flow is calculated by dividing 3,600 by that constant headway. It is usually given in terms of vehicles per hour of green (vphg).

For two situations, the headway field was left blank because of irregularities in traffic flow. One was when the first vehicle stopped beyond the stop bar. In that case, the headways for the first two vehicles were left blank. The other situation was when an interruption code was encountered. In that case, headways were left blank for the interrupted vehicle and the following vehicle.

Another program was written and executed that checked for various types of errors in the data. This program discovered several minor coding errors, which were corrected prior to subsequent analysis.

A summary program was written to allow the data to be summarized. That program was designed so that the data included in each summary could be limited, based upon values of all important variables. The program also allowed each summary to be presented by various categories, with the categories defined as desired. A sample of output from that program is shown in Figure 7.

An earlier version of the program, having different output, was also used in the initial stage of analysis. A sample of output from this program is shown in Figure 8.

Analysis was performed by limiting values of all but one important variable, allowing that variable to vary, and observing the effect of that variance on saturation flow. The analysis was a careful, step-by-step process, with results of each step affecting limitations applied to successive steps. Where necessary, assumptions were made and later verified. If assumptions were found to be invalid, certain steps of the analysis were repeated with necessary corrections. Additional data also were collected to fill in gaps that became apparent during analysis. The data file contained a total of approximately 47,000 headways, of which approximately 32,000 were collected in Lexington.

RESULTS

VEHICLE POSITION IN QUEUE

The first step in data analysis was to determine how average headway varied with vehicle position in the queue. Average headway should be highest for the first vehicle, decrease slightly for the next few vehicles, and then become fairly constant. This constant headway is used to calculate saturation flow. The purpose of this first step was to determine how many initial vehicles had to pass at the beginning of a green phase before headways became fairly constant. This goal was accomplished by obtaining a printout of average headway versus queue position.

For these summaries, data for which lane width was 10 to 15 feet and grade was from minus three to plus three percent were included. The first summary prepared was for through (non-turning) passenger cars in Louisville and Lexington. The results of that summary, presented in Figure 9, indicated that headways became fairly constant after the first three vehicles, i.e., beginning with vehicle number four. Another summary, for through passenger cars in all other cities, generated the same conclusion. A third summary, for commercial vehicles in all cities, indicated the first four vehicles had higher headways, and saturation flow began with the fifth. Based upon those results, and considering that passenger cars comprise the majority of most traffic streams, it was decided to define saturation flow as beginning with the fourth vehicle in the queue. All subsequent summaries used that definition.

LOCATION IN CITY AND PEDESTRIAN ACTIVITY

Each studied intersection was classified as being in a central business district (CBD), fringe area, outlying business district, or residential area. This classification used the following definitions from the Highway Capacity Manual (2).

1. Central business district -- That portion of a municipality in which the dominant land use is intense business activity.
2. Fringe area -- That portion of a municipality immediately outside the central business district in which there is a wide range in type of business activity, generally including small commercial, light industrial, warehousing, automobile service activities, and intermediate strip development, as well as some concentrated residential areas.
3. Outlying business district -- That portion of a municipality or an

area within the influence of a municipality, normally separated geographically by some distance from the central business district and its fringe area, in which the principal land use is for business activity.

4. Residential area -- That portion of a municipality, or an area within the influence of a municipality, in which the dominant land use is residential development, but where small business areas may be included.

Level of pedestrian activity was observed to be closely related to location in city. Each studied intersection was classified as having light, moderate, or heavy pedestrian activity. The pedestrian activity rating was subjective and used the following guidelines. Locations rated as having heavy pedestrian activity had fairly continuous pedestrian flows, with pedestrians crossing on every cycle and frequent interference with turning vehicles. These locations generally had marked crosswalks and pedestrian signals. Moderate pedestrian activity was characterized by less continuous flows, with pedestrians crossing on some, but not all, cycles, and interference with turning vehicles occuring less frequently. Light pedestrian activity was coded for locations with few pedestrians, with most cycles going unused for pedestrian crossing, and with rare or nonexistant interference with turning vehicles.

Data were not included in the analysis for any vehicle which encountered actual pedestrian interference. The purpose of this analysis was to determine the effect of the general level of pedestrian activity, exclusive of lost time due to individual cases of interference.

Due to the strong relationship between pedestrian activity and location in city, these two factors were combined for analysis. To control for effects of other significant variables (which are discussed in following sections), the summary excluded cities with populations under 20,000, and included only through and left-turning passenger cars, lane widths from 10 to 15 feet, grades from minus three to plus three percent, speed limits of 35 to 45 miles per hour (mph), and approaches with no parking within 200 feet of the stop bar.

Results of the summary are presented in Table 1, which shows saturation flow for various combinations of location in city and level of pedestrian activity. Saturation flow was lowest for locations with heavy pedestrian activity and for locations in the CBD, although the single lowest value was for heavy pedestrian activity in fringe areas. There was little difference between values for fringe areas, outlying business districts, and residential areas. There also was little difference between values for light and moderate pedestrian activity levels. Locations with heavy pedestrian activity had saturation flow levels about four percent lower than other locations, as did locations in the CBD.

CITY SIZE

To determine effects of city size on saturation flow, the eight cities for which data had been collected were classified into four categories based upon population. Populations of those cities, the method of classification, and average population for each category are shown in Table 2. Included in that summary were lane widths of 10 to 15 feet, grades from minus three to plus three percent, and and speed limits of 35 to 45 mph. The summary was limited to through and left-turning passenger cars, and data taken in the central business district were excluded. Also excluded were locations with heavy pedestrian activity and approaches with parking within 200 feet of the stop bar.

Results of that summary are also included in Table 2. Saturation flow increased with increasing population, with levels in Louisville 21 percent higher than in the lowest population category. For the top three population categories, the increase amounted to an eight percent difference over a population range of 30,000 to 500,000. For the lowest population category, saturation flow levels decreased sharply, with a value 10 percent lower than for the next lowest category. Thus, it appears that for populations of about 20,000 to 500,000, population has a moderate effect on saturation flow. However, for populations under 20,000, saturation flow values decrease substantially. Unless otherwise noted, all following summaries excluded cities in the lowest population category.

VEHICLE TYPE AND TURNING MANEUVER

Saturation flow levels have been noted to vary significantly as a function of vehicle type and turning maneuver. Right- and left-turning vehicles were identified as such during data collection. All left turns observed were made during protected phases, so opposing traffic was not a factor. Vehicle types other than those included in the passenger car category (which included pickup trucks and vans) were also noted on the data sheet. A single-unit commercial vehicle was defined as any single-unit truck that had more than four tires. A combination commercial vehicle was defined as any regular combination truck or truck with a trailer. Data for this analysis were limited to locations with lane widths of 10 to 15 feet, grades from minus three to plus three percent, speed limits of 35 to 45 mph, and no parking on the approach within 200 feet of the stop bar. In addition, locations with heavy pedestrian activity, locations in a CBD, and cities with populations under 20,000 were excluded from the analysis. For right-turning vehicles, only locations with turning radii from 25 to 44 feet were included.

A summary of data collected, by vehicle type and turning maneuver, is in Table 3. Headways were lowest for through vehicles and highest for right-turning vehicles. Combination commercial vehicles had the highest average headways (lowest saturation flow levels) of any vehicle type. Values for single-unit commercial vehicles and buses were similar. Motorcycles had the lowest average headway (highest saturation flow) of the vehicle types listed.

Typically, some type of equivalency value has been used to compare vehicle types and turning maneuvers. "Through car equivalent" (TCE) values, by vehicle type and turning maneuver, are in Table 4. All categories were related to the through passenger car category. For passenger cars, left turns had little effect, but right-turning passenger cars had a TCE value of 1.1. Combination commercial vehicles had the highest TCE values. One through combination commercial vehicle was equivalent to 2.0 through passenger cars. Turning combination commercial vehicles were equivalent to 2.4 through passenger cars. Through and left-turning motorcycles had the lowest TCE values, at about 0.8. Single-unit commercial trucks and buses had TCE values around 1.5.

GRADIENT

To determine effects of approach grade on saturation flow, computer summaries which categorized the data by grade were prepared. Each of these summaries included locations having lane widths from 10 to 15 feet and speed limits of 35 to 45 mph. Excluded were locations in a CBD or in a city having a population under 20,000. Also excluded were locations with heavy pedestrian activity or with parking on the approach within 200 feet of the stop bar. The first summary was for through and left-turning passenger cars. Results are shown in Table 5. Increasing grade decreased saturation flow, although the top grade category (grade greater than three percent) did not show the expected increase. For other categories, the magnitude of the effect was about a 1.1 percent decrease in saturation flow for every one percent increase in grade. It is difficult to explain the results for grades greater than three percent, as they were approximately equal to the results for grades between one and three percent. It was expected that saturation flow would drop for higher grades.

A second summary, including only through commercial vehicles and buses, was prepared. It was expected that grade would have a more profound effect on those vehicles than on passenger cars. Because of limited data for thoese vehicles, this summary was combined into three grade categories. Results are shown in Table 5. Again, positive grades did not show the expected effect, although negative grades did increase saturation flow substantially.

To attempt to quantify the effect of steep positive grades, additional data were collected for a location with a positive grade of 6.5 percent. Results for this location were then compared to locations having similar characteristics but different grades. Only passenger cars were included. The resulting saturation flow was five percent lower for the steep grade than for flat grades.

LANE WIDTH

A summary of results of the lane width analysis is in Table 6. The summary excluded data taken in cities with populations under 20,000. Also excluded were locations in a CBD, locations with heavy pedestrian activity, and locations with parking on the approach within 200 feet of the stop bar. Included in the summary were approach grades of minus three to plus three percent and speed limits of 35 to 45 mph. Only through and left-turning passenger cars were included.

The analysis indicated than lane width did not have an effect on saturation flow for lane widths of 10 feet or more. For lane widths between nine and ten feet, a five percent reduction in saturation flow was found compared to lane widths of 10 or more feet. No lane widths below nine feet were observed. There was a slight unexplained reduction in saturation flow for lane widths greater than 15 feet. A similar analysis was performed with the limited data available for commercial vehicles, and no effect was found, even for lane widths below 10 feet.

TURNING RADIUS

Summaries were prepared, categorized by turning radius, to determine the effect of turning radius on right-turning vehicles . The summaries were limited to non-CBD locations in cities with populations of 20,000 or more having lane widths of 10 to 15 feet, grades from minus three to plus three percent, and speed limits of 35 to 45 mph. Excluded from the summary were locations with heavy pedestrian activity or with parking on the approach within 200 feet of the stop bar. The initial summary was limited to right-turning passenger cars. Results of that summary are in Table 7. Locations having turning radii less than 25 feet had saturation flows approximately eight percent lower than locations having turning radii in the 25-to-44 foot range. Increasing the right-turning radius above 44 feet increased the saturation flow by about two percent.

Turning radius was expected to have a greater effect on large vehicles, such as trucks and buses, than on passenger cars. Therefore, a second summary was prepared for right-turning commercial vehicles and buses. However, there were too few of those vehicles in the data to allow a meaningful analysis. Turning radii for left turns generally are sufficiently large so that they are not a factor. An exception is left turns from a one-way street onto another one-way street (from the left-most lane), for which the radius may indeed be a factor. For this case, the results for right turns could be applied to left turns as well.

PARKING

A summary was prepared to compare saturation flows for locations with and without parking. The objective was to determine whether parking on an approach affected the saturation flow level on that approach. Only parking on the right side of an approach was included, and parking had to be within 200 feet of the stop bar. Cities with populations under 20,000 were excluded, and only the right-most lane on each approach was considered as having parking when parking existed. Because of the limited number of locations where parking was allowed, this summary was limited to CBD locations having heavy pedestrian activity, grades from minus three to plus three percent, speed limits of 35 mph, and no parking on the left side of the approach. The summary included lane widths of 17 to 22 feet, which included the width used for parking. This is equivalent to lane widths of about 10 to 15 feet not including the width used for parking. Only through passenger cars were included. Results of that summary indicated no effect due to parking, with an average headway of about 2.74 in each case.

Additional comparisons were made of locations which were similar except that one had parking and the other did not. Two such comparisons were made, each comparing an approach having marked parking adjacent to an 11- to 13-foot lane with an approach having a similar lane width but no parking. Neither of these comparisons showed any effect of parking.

Apparently, for lane widths considered here, it was possible to provide parking without interfering with traffic flow. If parking were located where it decreased the effective width of the adjacent lane to less than 10 feet, it should cause a decrease in saturation flow. However, sufficient data are not available to validate this assumption.

APPROACH WIDTH OR WIDTH OF ADJACENT LANE

The effect of the approach width or, more specifically, the width of an adjacent lane on the saturation flow level of a particular lane was analyzed. In this analysis, data at approaches having two through lanes were summarized to determine whether width of the adjacent lane had an effect on saturation flow of a through lane. In this report, a through lane is defined as any lane from which a vehicle may proceed straight through the intersection, without turning. This may or may not be a through-only lane. The width of the adjacent lane was not coded in the data, but had to be obtained indirectly. This was achieved by limiting the summaries to locations with two through lanes, restricting the lane width, and allowing the total through width to vary. The data were controlled for vehicle type, turning maneuver, city size, location in city, grade, speed limit, pedestrian activity, and parking in order to limit the effects of those factors.

An initial analysis was performed where the lane width was controlled to be from 10 to 15 feet. Saturation flow dropped slightly when the total approach width of two through lanes was below 22 feet. However, widths of the two individual lanes could not be estimated from that summary. A second analysis was conducted, as shown in Table 8, in which lane width was limited to between 11.5 and 12.5 feet. That allowed the approximate width of the adjacent lane to be determined. The summary indicated that an adjacent lane width above 10 feet did not affect saturation flow for a traffic lane. No data were available for a 12-foot lane with an adjacent lane less than 10 feet wide. A search detected two locations with lane widths of 11 and 13 feet that had adjacent lanes between 9 and 10 feet wide. The average of 118 headways at those locations yielded an average headway of 2.22 seconds, which was not significantly above that observed for lanes with wide adjacent lanes.

This analysis indicated no substantial effect of approach width or, more specifically, width of an adjacent lane on the saturation flow level of a particular lane.

TYPE OF LANE

This segment of the analysis first analyzed saturation flows on different types of through lanes. The summary included non-CBD locations in cities with populations over 20,000, with lane widths of 10 to 15 feet, approach grades from minus three to plus three percent, and speed limits of 35 to 45 mph. Heavy pedestrian flows were excluded, as were approaches with parking within 200 feet of the stop bar. Only passenger cars were included. Results of the summary for through lanes are in Table 9. Saturation flows were calculated for; single through-only lanes (the only lane on an approach which is strictly for through movements), the left and right of dual through-only lanes, and each lane of triple through-only lanes. The single through-only lane had the lowest saturation flow, followed by the right dual, left dual, right triple, left triple, and middle triple.

This suggests that saturation flow is affected by the amount of "friction" between adjacent lanes, with friction depending on relative speeds and vehicle maneuvers. The single through-only lane has the highest friction, since it is adjacent to either turning lanes, combination through and turning lanes, opposing traffic, or the roadway edge. Friction is reduced by going to multiple through-only lanes. the lowest friction is for the middle of three through-only lanes.

A similar summary was prepared for saturation flow in through-only lanes as a function of number of through lanes. Number of through lanes means the number of lanes on the approach from which through movements are allowed, including combination through and turning lanes. Results are also in Table 9. These results effectively demonstrate the increase in saturation flow in a through-only lane caused by providing additional through lanes. A four percent increase occured when one additional through lane was provided, while an eight percent increase resulted from providing two additional through lanes.

A summary was also prepared for single versus multiple left-turn lanes. As shown in Table 9, the effect of multiple lanes for left turns was opposite that for through vehicles. Saturation flow for single left-turn lanes was higher than for either of the dual left-turn lanes, with the lowest being for the left of the duals. Saturation flow was five percent lower on the left dual than on the right dual, while average saturation flow per lane for duals was five percent lower than for a single lane. This effect was not totally unexpected. Drivers are often uncomfortable with dual turning lanes. This type of lane is fairly uncommon and drivers are sometimes unsure of the proper path to follow. That attitude would tend to increase headways and decrease saturation flow.

TIME OF DAY (PEAK OR NON-PEAK HOURS)

To determine the effect of time of day, or peak and non-peak traffic volumes, a comparison was made of data collected during loaded and non-loaded cycles. A loaded cycle is one that is fully utilized; that is, one for which the queue does not fully discharge during the green-plus-yellow phase.

Factors noted to affect saturation flow levels were controlled. The analysis excluded data collected in small cities, in CBD's, or at locations having heavy pedestrian activity. The analysis was limited to through and left-turning passenger cars, lane widths between 10 and 15 feet, approach grades between plus three and minus three percent, speed limits of 35 to 45 mph, and no parking on the approach within 200 feet of the stop bar.

It was thought that saturation flow levels might be higher during peak conditions (periods with loaded cycles). However, results in Table 10 show no major difference between saturation flow levels for loaded and non-loaded cycles. Loaded cycles had about a one percent higher saturation flow level when compared to non-loaded cycles.

SPEED LIMIT

A summary was prepared to determine the effect of approach speed limit on saturation flow. Data were controlled for city size, location in city, lane width, grade, pedestrian activity, and parking. Only through and left-turning passenger cars were included. Results of that summary are in Table 11. As speed limit increased, saturation flow also increased. However, the effect was relatively small, amounting to approximately four percent over the range from 35 to 55 miles per hour.

WEATHER OR ROAD SURFACE CONDITIONS

Data collected on one approach during a light rain when the pavement was wet were compared to data collected on the same approach during dry conditions. The data were collected on a through-only lane, and only passenger cars were included in the analysis. During dry conditions, an average headway of 2.17 seconds was noted (representing almost 500 headways). That compared well with the overall average headway for through passenger cars (2.19 seconds). The average of approximately 300 headways collected during a light rain (wet-pavement conditions) was 2.20 seconds. That represents a very slight increase (1.4 percent) in average headway, which would result in a corresponding slight decrease in saturation flow. The difference is too small to consider this a factor affecting saturation flow. Of course, a heavy rain, reducing visibility, would be expected to reduce saturation flow levels, as would snow or ice. However, these are unusual conditions which generally would not be used in routine design procedure.

LIGHT CONDITIONS

To evaluate the effect of light conditions on saturation flow, data were collected at a selected location during both daylight and darkness. Only through passenger cars were included. Average saturation flow levels observed were 1,659 vphg during daylight and 1,558 vphg during darkness. This represents a six-percent difference. The reduction indicates there is some alteration of driver behavior due to light conditions, with slightly lower saturation flow levels during darkness.

While this is an interesting phenomenon, it may not have any significant bearing on intersection design. It would be important if an intersection was expected to handle peak volumes of traffic during darkness.

RECOMMENDATIONS

Results from data analyses identified several factors that significantly influenced saturation flow. To determine an accurate saturation flow value for a specific intersection, approach, and lane, appropriate adjustment factors can be applied to a "base" saturation flow value. The "base" value applies to passenger cars proceeding straight through an intersection under generally typical conditions. That value, on a per-lane basis, was shown to be 1,650 vehicles per hour of green (vphg).

The following formula is recommended for use in estimating an appropriate saturation flow value for a specific lane on an approach to an intersection:

$$S = Sb(Flp)(Fc)(Fvt)(Fg)(Fw)(Fr)(Ft)(Fs)(Fd)$$

in which S = saturation flow for a specific intersection approach lane (vehicles per hour of green (vphg))
Sb = base saturation flow value (1,650 vphg),
Flp = adjustment factor for location in city and level of pedestrian activity,
Fc = adjustment factor for city population,
Fvt = adjustment factor for vehicle type and turning maneuver,
Fg = adjustment factor for gradient,
Fw = adjustment factor for width of lane,
Fr = adjustment factor for turning radius for right-turning vehicles,
Ft = adjustment factor for type of lane
Fs = adjustment factor for speed limit, and
Fd = adjustment factor for light conditions.

Following is a discussion of the recommended methods for determining appropriate adjustment factors.

Flp (adjustment factor for location in city and level of pedestrian activity)
Flp = 0.96 for locations with heavy pedestrian

activity.

Flp = 0.97 for CBD locations with light or moderate pedestrian activity.

Flp = 1.00 for all other locations.

Fc (adjustment factor for city population)

Fc = 0.87 for population under 10,000.

Fc = 0.92 for population of 10,000 to 19,999.

Fc = 0.97 for population of 20,000 to 99,999.

Fc = 1.00 for population of 100,000 to 249,999.

Fc = 1.05 for population of 250,000 to 500,000.

Fvt (adjustment factor for vehicle type and turning maneuver)

Fvt = 100/((P1)(T1) + (P2)(T2)+ ...+(P10)(T10))

in which P1 through P10 are the percentages of different vehicle types in the anticipated or actual traffic stream for the lane being considered and T1 through T10 are the corresponding "through car equivalents" for these vehicle types. P1 through P10 must total to 100 percent.

P1: through passenger cars, T1 = 1.00
P2: left-turning passenger cars, T2 = 0.98
P3: right-turning passenger cars, T3 = 1.12
P4: through single-unit commercial vehicles, T4=1.36
P5: left-turning single-unit commercial vehicles, T5=1.57
P6: right-turning single-unit commercial vehicles, T6=1.71
P7: through combination commercial vehicles, T7=2.02
P8: turning combination commercial vehicles, T8=2.41
P9: all buses, T9=1.55
P10: all motorcycles, T10=0.85

Fg (adjustment factor for approach gradient)

Fg = 1 - 1.1(Grade/100) for downhill approach.

Fg = 1 - 0.5(Grade/100) for uphill approach.

(Note: Enter grade in percent, negative for downhill approach).

Fw (adjustment factor for width of lane)

Fw = 0.95 for lane width of 9.0 to 9.9 feet.

Fw = 1.00 for lane widths of 10 or more feet.

Fr (adjustment for turning radius. Equation given is for effect on right-turning vehicles. For left turns from one one-way street to another, switch the terms "right-turning" and "left-turning" in the equation.)

Fr = (percent through and left-turning vehicles)/100 + Frd(percent right-turning vehicles)/100

in which Frd = 0.93 for turning radius less than 25 feet.

Frd = 1.00 for turning radius of 25 to 44 feet.

Frd = 1.03 for turning radius of 45 or more feet.

Ft (adjustment factor for type of lane)

Ft = 1.02 for a single left-turn-only lane.

Ft = 0.95 for left of dual left-turn-only lanes.

Ft = 1.10 for a through-only lane with through lanes on both sides.

Ft = 1.05 for left or right of triple through-only lanes.

Ft = 0.96 for through lane that is the only through lane on approach.

Ft = 1.00 for all other lane

types.

Fs (adjustment for speed limit)

Fs = 1.03 for speed limit above 45 mph.

Fs = 1.00 for speed limit of 45 mph or less.

Fd (adjustment for darkness)

Fd = 0.94 for locations where peak volumes are expected to occur during darkness.

Fd = 1.00 for all other locations.

While this equation may appear formidable at first, it is quite easy to use. This is illustrated in Appendix B, which examines case studies and compares measured and predicted saturation flows.

IMPLEMENTATION

The methodology developed in this study will allow the determination of saturation flow values representative of conditions at a given intersection. These saturation flow values provide basic input needed in the calculation of intersection capacity. This type of information is also needed as input when using computer models which simulate and optimize signal systems. Accurate saturation flow values are necessary in order to properly utilize these computer models. For example, portions of the data collected in this study were used as input into the TRANSYT, NETSIM, and SIGOP computer simulation models when they were used to simulate traffic flow in Lexington, Kentucky.

;ure 1. Variation of Discharge Rate of a Queue with Time.

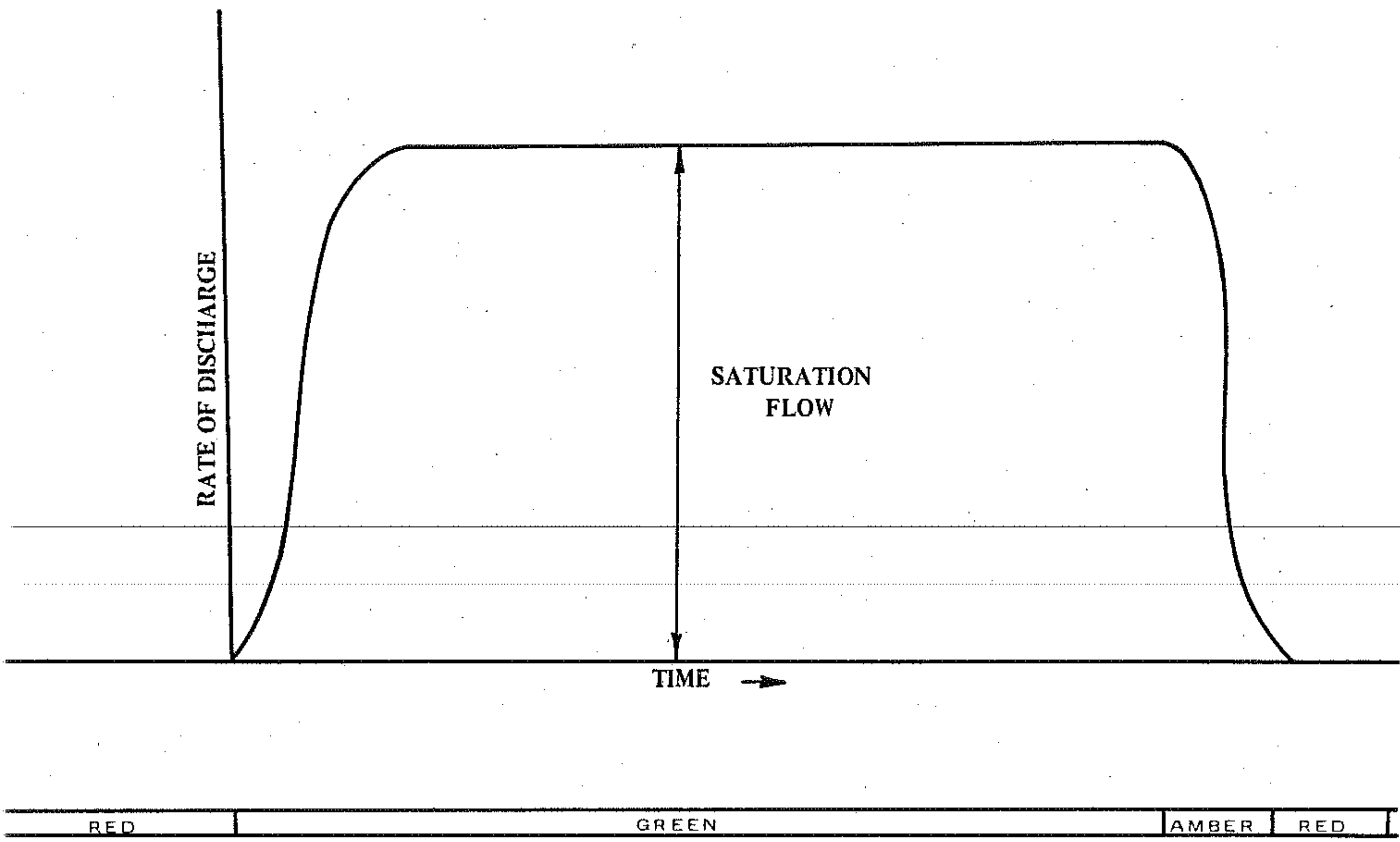

Figure 2. Saturation Flow Approach Data Sheet.

SATURATION FLOW
APPROACH DATA SHEET

City Name ________ Street Name ________ Approach Dir. ________

Cross-Street Name ________ Approach (+ or -) Grade ________

One-way or two way Speed limit ________

Location: (CBD, Fringe, OBD, Res.) Ped. Act.: (L, M, H)

Distance back to nearest parking on approach, measured from stop bar:

Right side ________
Left side (1 way street only) ________

Lane information (left to right, each lane)

Number	Type	Width	Gutter	Turning Radius	Distance (Stop bar to Intersection)
1	____	____	____	____	________
2	____	____	____	____	________
3	____	____	____	____	________
4	____	____	____	____	________

Total number of lanes on approach ____

Total width ____

SKETCH

SATURATION FLOW DATA SHEET

Date - ____________ Time - ____________ City - ____________

Street - ____________ Cross-Street - ____________ Approach - ____________

Comments____________ Y - ____________ If fixed time - G - ________ C - ________ Lane - ________

Cycle Number

	1		2		3		4		5		6		7		8		9		1
n	t	d	t	d	t	d	t	d	t	d	t	d	t	d	t	d	t	d	t
1																			
2																			
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			
11																			
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20																			
21																			
22																			
23																			
24																			
25																			
--																			
L																			
G + Y																			

Figure 3. Saturation Flow Data Sheet.

Figure 4. Saturation Flow Data Collection Coding.

SATURATION FLOW DATA COLLECTION CODING

Lane Type Code

1--Left Turn Only	5--Left Turn, Right Turn, and Straight
2--Right Turn Only	6--Straight Only (One Lane)
3--Left Turn and Straight	7--Straight Only (Left of Two Such Lanes)
4--Right Turn and Straight	8--Straight Only (Right of Two Such Lanes)

N--Vehicle Number in Queue

T--Time from Start of Green to Screen Line (Rear Axle over Stop Bar)

D--Description

TURNING	TYPE	INTERRUPTION
L--Left Turn	B--Bus	P--Pedestrian
R--Right Turn	M--Motorcycle	T--Bus Stopping
	S--Single Unit Truck	V--Stalled Vehicle
	C--Combination Truck	N--Back from Next Intersection
	I--Bicycle	O--Other (Specify)

L--Check If Cycle is Loaded

G--Green Time (Seconds) (If fixed time, note in comments)

Y--Yellow Time (Seconds)

C--Cycle Length (Seconds)

Figure 5. Saturation Flow Data Format.

HEADER RECORD

COLUMN	VARIABLE NAME	COMMENTS
1	CITY	City Code
2-3	INT	Intersection Code
4	APP	Approach Code
5	LANE	Lane Code
6		
7	LOC	Location in Urban Area Code
8-11	WAPP	Width of Aproach (Feet)(e.g. 25.0)
12-15	WTHRU	Width of Through Lanes
16-19	WLANE	Width of Lane
20-22	WGUTTER	Width of Gutter (if applicable)
23		
24	NLANES	Number of Lanes
25	NTHRU	Number of Through Lanes
26-29	GRADE	Grade in Percent (e.g. +1.5)
30	ONETWO	One-way or Two-way Street Code
31-32	SPEED	Speed Limit (MPH)
33	PED	Pedestrian Activity Code
34		
35-37	PARKR	Distance to Parking (Ft)(Right Side)
38-40	PARKL	Distance to Parking (Ft)(Left Side)
41-42	TURNRAD	Turning Radii (Ft)(if applicable)
43-44	DISTB	Distance from Stop Bar to Intersection
45		
46-50	CYCLE	Cycle Length (sec) (if fixed) (to 0.1 sec)
51-53	YELLOW	Yellow Time in Seconds (to 0.1 sec)
60	HEADER	Header Card Flag (Code 1)

DATA RECORD

COLUMN	VARIABLE NAME	COMMENTS
1	CITY	City Code
2-3	INT	Intersection Code
4	APP	Approach Code
5	LANE	Lane Code
6-7	CYCLE	Cycle Number
8		
9-10	VNUM	Vehicle Number in Queue
11-15	TIME	Time from Start of Green
16		
17	VTURN	Turning Code
18	VTYPE	Type of Vehicle
19	INTCODE	Interruption Code
20		
21	LAST	Last Car of Cycle Flag(0-no; 1-yes)
22	LOADED	Loaded Cycle Flag (1-yes)
23	LASTC	Last Cycle for This Lane Flag (1-yes)
24		
25-28	GYPT	Green plus Yellow in (sec) (if loaded cycle)
29		
30-33	HEADWAY	Headway in Seconds (to 0.1 sec)
34		
35	WET	Wet Surface Flag (1-yes)(first vehicle in cycle)
36		
37	DARK	Darkness Flag (1-yes)(first vehicle in cycle)

Figure 6. Saturation Flow Coding Information.

SATURATION FLOW CODING INFORMATION

CITY

1--Louisville
2--Lexington
3--Owensboro
4--Bowling Green
5--Paducah
6--Nicholasville
7--Richmond
8--Hazard
9--Somerset

INTERSECTION

(listed separately by city)

APPROACH

1--Northbound Approach
2--Southbound Approach
3--Eastbound Approach
4--Westbound Approach

LANE

1--Left turn only
2--Right turn only
3--Left turn and straight
4--Right turn and straight
5--Left turn, Right turn and straight
6--Straight only
7--Straight only (left of two such lanes)
8--Straight only (right of two such lanes)

LOCATION IN CITY

1--Central Business District
2--Fringe Area
3--Outlying Business District
4--Residential Area

PEDESTRIAN ACTIVITY

1--Light   2--Medium   3--Heavy

TURNING

blank or 0 -- Normal   1--Left   2--Right

TYPE OF VEHICLE

blank or 0--Normal
1--Commercial
2--Single Unit Commercial
3--Combination Commercial
4--Bus
5--Motorcycle
6--Bicycle

INTERRUPTION

blank or 0--No interruption
1--pedestrian interfering with turning or going straight
2--backup from next intersection
3--bus stopping
4--stalled car
5--other

ONE-WAY OR TWO-WAY

1--One-Way
2--Two-Way

Figure 7. Sample of Summary Program Output.

```
                SATURATION FLOW DATA ANALYSIS

ALL CITIES
URBAN LOCATIONS INCLUDED =  2  3  4
LANE WIDTHS INCLUDED = 10.0 - 15.0
GRADES INCLUDED = -3.0 -  3.0
SPEED LIMITS INCLUDED = 35 - 45
PEDESTRIAN ACTIVITY CODES INCLUDED = 1  2
RIGHT PARKING DISTANCES INCLUDED = 0 -  0
LEFT PARKING DISTANCES INCLUDED = 0 -  0
VEHICLE TURNING CODES INCLUDED = 0  1
VEHICLE TYPES INCLUDED = 0  0  0  0
INTERRUPTION CODES INCLUDED = 0


DESCR. OF CATEGORIES:  1=LOUISVILLE
                       2=LEXINGTON
                       3=BOW GR,PADUCAH,RICHMOND
                       4=NICH,HAZARD,SOMERSET
                       5=OTHER
                       6=
                       7=

CATEGORY=1 AVG HDWYS FOR VNUM=1,2,3,ETC.=2.84 2.50 2.31
AVG HDWY FOR VNUM>3 IS 2.08  AVG LOST TIME=1.43
TOTAL HDWYS AVERAGED= 1259

CATEGORY=2 AVG HDWYS FOR VNUM=1,2,3,ETC.=2.82 2.52 2.41
AVG HDWY FOR VNUM>3 IS 2.18  AVG LOST TIME=1.21
TOTAL HDWYS AVERAGED=13056

CATEGORY=3 AVG HDWYS FOR VNUM=1,2,3,ETC.= 3.26 2.72 2.58
AVG HDWY FOR VNUM>3 IS 2.25  AVG LOST TIME= 1.74
TOTAL HDWYS AVERAGED= 2067

CATEGORY=4 AVG HDWYS FOR VNUM=1,2,3,ETC.= 2.76 2.52 2.54
AVG HDWY FOR VNUM>3 IS 2.51  AVG LOST TIME=1.93
TOTAL HDWYS AVERAGED= 2027

CATEGORY=5 NO DATA PROCESSED.

CATEGORY=6 NO DATA PROCESSED.

CATEGORY=7 NO DATA PROCESSED.
```

Figure 8. Sample Output of Summary Program Variation.

```
          SATURATION FLOW DATA ANALYSIS

CITIES INCLUDED = 1  2  0  0
LANE WIDTHS INCLUDED = 10.0 - 15.0
GRADES INCLUDED - -3.0 - 3.0
VEHICLE TURNING CODES INCLUDED = 0  0
VEHICLE TYPES INCLUDED = 0  0  0
INTERRUPTION CODES INCLUDED = 0

                              NUMBER OF
VNUM        AVG HDWY       VALUES AVERAGED
 1             2.9              1648
 2             2.6              1581
 3             2.4              1586
 4             2.3              1583
 5             2.3              1493
 6             2.2              1401
 7             2.2              1307
 8             2.1              1192
 9             2.2              1104
10             2.1               987
11             2.2               891
12             2.2               779
13             2.2               667
14             2.1               569
15             2.1               478
16             2.1               396
17             2.1               327
18             2.1               283
19             2.2               225
20             2.1               175
21             2.1               137
22             2.2               103
23             2.2                79
24             2.1                49
25             2.3                32
26             2.6                 7
27             1.4                 1
28             0.0                 0
29             0.0                 0
30             0.0                 0
31             0.0                 0
32             0.0                 0
33             0.0                 0
34             0.0                 0
35             0.0                 0
36             0.0                 0
37             0.0                 0
38             0.0                 0
39             0.0                 0
 AVERAGE HEADWAY WHEN VNUM > 4 = 2.2
 TOTAL NUMBER OF:
 INTERSECTIONS:      45
 APPROACHES          71
 LANES               99
 TOTAL NUMBER OF HEADWAYS = 19080
 END LOST TIME
 AVERAGE TIME = 1.5
 NUMBER OF VALUES AVERAGED = 251
```

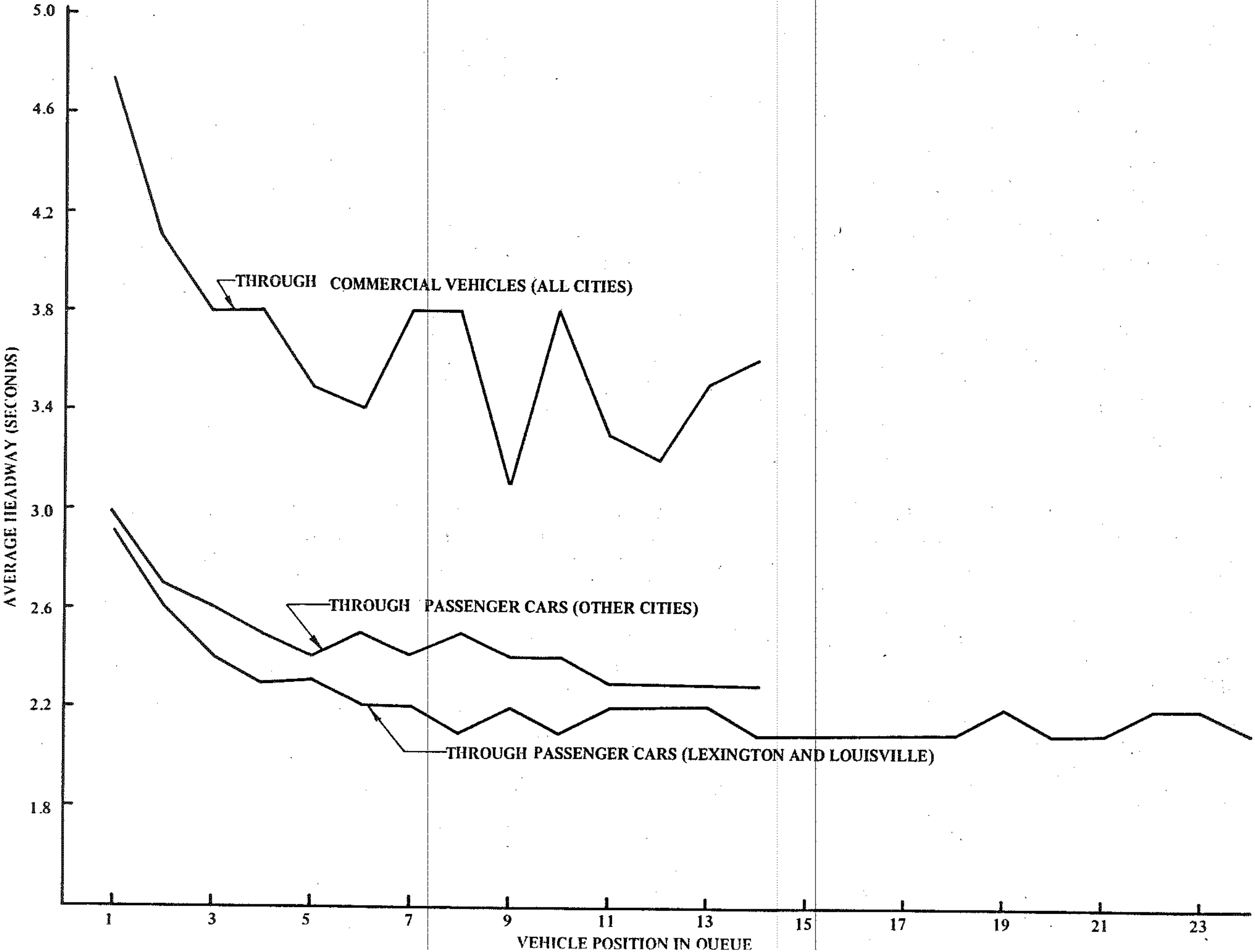

Figure 9. Average Headway Versus Vehicle Position in Queue.

TABLE 1. EFFECT OF LOCATION IN CITY AND LEVEL OF PEDESTRIAN ACTIVITY ON SATURATION FLOW

LOCATION IN CITY		LEVEL OF PEDESTRIAN ACTIVITY: LIGHT	MODERATE	HEAVY	ALL LEVELS
Central Business District	Number of Headways Averaged	0	22	1,173	1,195
	Average Headway (seconds)	DNA	2.25	2.26	2.26
	Saturation Flow (vphg)*	DNA	1,600	1,593	1,593
Fringe Area	Number of Headways Averaged	1,967	2,741	569	5,277
	Average Headway (seconds)	2.22	2.20	2.32	2.22
	Saturation Flow (vphg)*	1,622	1,636	1,552	1,621
Outlying Business District	Number of Headways Averaged	9,859	888	0	10,747
	Average Headway (seconds)	2.16	2.18	DNA	2.16
	Saturation Flow (vphg)*	1,667	1,651	DNA	1,665
Residential Area	Number of Headways Averaged	927	0	0	927
	Average Headway (seconds)	2.21	DNA	DNA	2.21
	Saturation Flow (vphg)*	1,629	DNA	DNA	1,629
ALL AREA	Number of Headways Averaged	12,753	3,651	1,742	18,146
	Average Headway (seconds)	2.17	2.20	2.28	2.19
	Saturation Flow (vphg)*	1,657	1,640	1,579	1,646

* Vehicles (per lane) per hour of green.

TABLE 2. EFFECT OF CITY SIZE ON SATURATION FLOW

CATEGORY	CITIES	POPULATION	AVERAGE POPULATION	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
1	Louisville	490,100*	490,100	1,259	2.08	1,731
2	Lexington	204,200	204,200	13,056	2.18	1,651
3	Bowling Green Paducah Richmond	40,400 29,800 21,700	30,600	2,067	2.25	1,600
4	Somerset Nicholasville Hazard	10,600 9,800 5,400	8,600	2,027	2.51	1,434

*Louisville metropolitan area.

TABLE 3. EFFECT OF VEHICLE TYPE AND TURNING MANEUVER ON SATURATION FLOW

TURNING MANEUVER		VEHICLE TYPE: PASSENGER CAR	SINGLE-UNIT COMMERCIAL	COMBINATION COMMERCIAL	BUS	MOTORCYCL
THROUGH	Total Headways Averaged	13,495	300	122	19	56
	Average Headway (Seconds)	2.19	2.98	4.42	3.39	1.87
	Saturation Flow (VPHG)	1,644	1,208	814	1,062	1,925
LEFT-TURNING	Total Headways Averaged	2,890	38	12	4	9
	Average Headway (Seconds)	2.14	3.44	5.27	3.42	1.81
	Saturation Flow (VPHG)	1,682	1,047	683	1,053	1,989
RIGHT-TURNING	Total Headways Averaged	973	14	0	0	1
	Average Headway (Seconds)	2.46	3.74	DNA	DNA	1.60
	Saturation Flow (VPHG)	1,463	963	DNA	DNA	2,250

TABLE 4. THROUGH CAR EQUIVALENTS (TCE) BY VEHICLE TYPE AND TURNING MANEUVER

	(THROUGH-CAR EQUIVALENTS (TCE)		
VEHICLE TYPE	THROUGH	LEFT-TURNING	RIGHT-TURNING
Passenger Car	1.0	1.0	1.1
Single-Unit Commercial	1.4	1.6	1.7
Combination Commercial	2.0	2.4	2.4*
Bus	1.6	1.6	1.7*
Motorcycle	0.9	0.8	1.0*

* Because of insufficient data, this value is an estimate. An analysis in which some of the data limitations were omitted was used to obtain these estimates.

TABLE 5. EFFECT OF GRADIENT ON SATURATION FLOW

GRADE (PERCENT)	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
	PASSENGER CARS		
Less than -3	1,481	2.09	1,719
-3 to -1.1	4,154	2.12	1,698
-1 to +1	10,763	2.19	1,644
+1.1 to +3	1,465	2.23	1,614
Greater than +3	798	2.22	1,622
	COMMERCIAL VEHICLES AND BUSES		
Less than -1	88	3.10	1,160
-1 to +1	361	3.48	1,034
Greater than +1	51	3.48	1,035

TABLE 6. EFFECT OF LANE WIDTH ON SATURATION FLOW

LANE WIDTH (FEET)	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
9 - 9.9	858	2.29	1,572
10 - 10.9	2,839	2.16	1,667
11 - 12.9	11,089	2.18	1,651
13 - 14.9	2,454	2.18	1,651
15 or More	680	2.21	1,629
10 - 14.9	16,382	2.18	1,654
10 or More	17,062	2.18	1,653

TABLE 7. EFFECT OF TURNING RADIUS ON SATURATION FLOW OF RIGHT-TURNING PASSENGER CARS

RADIUS (FEET)	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
Less than 25	321	2.66	1,354
25 to 44	973	2.46	1,465
45 or more	180	2.40	1,500

TABLE 8. EFFECT OF APPROACH WIDTH (SPECIFICALLY, THE WIDTH OF AN ADJACENT LANE) ON SATURATION FLOW

LANE WIDTH 11.5-12.5 FEET

WIDTH OF TWO THROUGH LANES (FEET)	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)	APPROXIMATE WIDTH OF ADJACENT LANE (FEET)
20 - 21.9	0	*	*	8-10
22 - 23.9	2,722	2.15	1,674	10-12
24 - 25.9	2,784	2.23	1,614	12-14
26 - 27.9	765	2.22	1,622	14-16
28 or More	0	*	*	16 or More

* No data available in this category. However, data for two through lanes 11 and 13 feet wide, respectively, with an adjacent lane between 9 and 10 feet wide were summarized. The average of 118 headways was 2.22 seconds, for a saturation flow level of 1,622 vphg.

TABLE 9. EFFECT OF TYPE OF LANE ON SATURATION FLOW

TYPE OF LANE	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
Through-Only (Only One on Approach)	2,785	2.21	1,629
Left of Two Through-Only Lanes	1,873	2.17	1,659
Right of Two Through-Only Lanes	3,758	2.19	1,644
Left of Three Through-Only Lanes	224	2.05	1,756
Middle of Three Throuth-Only Lanes	234	1.98	1,818
Right of Three Through-Only Lanes	222	2.10	1,714
One Through Lane on Approach*	146	2.28	1,579
Two Through Lanes on Approach*	7,979	2.19	1,644
Three Through Lanes on Approach*	971	2.12	1,698
Single Left-Turn-Only Lanes	1,995	2.09	1,722
Left of Two Left-Turn-Only Lanes	149	2.26	1,593
Right of Two Left-Turn-Only Lanes	406	2.14	1,682

* Data are for through-only lanes.

TABLE 10. EFFECT OF LOADED CYCLES ON SATURATION FLOW

	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
Loaded Cycles	4,375	2.16	1,667
Non-Loaded Cycles	18,057	2.18	1,651

TABLE 11. EFFECT OF SPEED LIMIT ON SATURATION FLOW

SPEED LIMIT (MPH)	TOTAL HEADWAYS	AVERAGE HEADWAY (SECONDS)	SATURATION FLOW (VPHG)
35	4,614	2.19	1,644
40	1,042	2.15	1,674
45	10,726	2.17	1,659
50	489	2.13	1,690
55	391	2.11	1,706

APPENDIX A

RESULTS OF LITERATURE REVIEW

MEASUREMENT OF SATURATION FLOW

Measurement of saturation flow involves measuring times from the beginning of the green phase until certain vehicles cross a reference line or screenline. A basic question concerns what screenline to use. In one paper, four alternate screenlines for measuring queue discharge headways at signals were compared (3). Screenline positions were the following:

1. the stopped front wheels of the first vehicle,
2. the stop line,
3. the crosswalk line (either the upstream or downstream line), and
4. the entry to the intersection.

For the first three alternate screenlines, the time could be recorded when either the front or rear wheels crossed the screenline. For the fourth alternative, the time would be recorded when the front of the vehicle reached the intersection. Use of the fourth screenline was recommended. Although all of these positions have been used in various studies, the most common screenline position is the stop line (stop bar).

Equipment used to collect data has varied. The most common procedure involved using a stopwatch. Other devices used include movie cameras (4), tape recorders (5), chart recorders (3), or automatic recorders (5). An automatic recorder may be coupled to a detector for each lane and to a light sensitive resistance in front of the green signal that records the start and finish of phases.

Several methods have been used to record and analyze data. Following is a sample of procedures that have been used.

1. A method presented in a 1979 report from Australia uses the number of vehicles departing from the queue after the first five seconds and the saturation period (period until the queue is cleared) to calculate saturation flow (6). Saturation flow for several cycles is calculated by dividing the average number of departures during the saturation period, excluding the initial five-second interval, by the average saturation period minus five seconds.
2. In a Road Research Laboratory method, a stopwatch is started when the signal changes to green, and the numbers of vehicles crossing the stop bar in regular time intervals (such as five seconds) are recorded until the end of the green. Duration of green and time at which saturation flow ceased are noted (5).
3. A method of obtaining small samples of accurate data used a pen recorder to record times (to the nearest 0.1 sec) at which vehicles crossed a tape switch placed across the roadway (5).
4. One procedure involved starting a stopwatch as the rear wheels of the first vehicle crossed the stop bar and stopping it as the last vehicle in the platoon crossed the stop bar with its rear wheels (7). The elapsed time was divided by the number of vehicles, less one, to determine average headway. Dividing 3,600 by the average headway (in seconds) yields saturation flow (in vehicles per hour of green).
5. In another procedure, a stopwatch was started as the third vehicle's rear bumper cleared the stop bar and stopped as the rear bumper of the last vehicle in the queue reached the stop bar. That time, divided by the number of vehicles minus three, provided the average headway.
6. Only cycles having 10 or more seconds of saturation flow were counted in one procedure (8). One second was subtracted per phase from the total time to account for the time lost in starting-up less the reaction time of the observer having the stopwatch.
7. The time required, after the signal turned green, for a

certain number of vehicles to cross the stop bar has also been used to calculate saturation flow (9, 10).

8. Cameras have been used to obtain the headway of each vehicle.

Saturation flow is generally measured in units of vehicles (per lane) per hour of green (vphg). Typical base values before adjustment are in the range of 1,500 to 1,700 vphg.

ADJUSTMENT FACTORS FOR SATURATION FLOW

Several factors have been observed to affect saturation flow. Following is a summary of factors found to affect saturation flow and methods used by various researchers to make appropriate adjustments.

Vehicle Position in Queue (Queue Length) -- Queue discharge headway is generally considered constant for all vehicles after some initial start-up delay that typically involves the first three to six vehicles. However, some data suggest that headways might again increase after a certain length of green time. One study concluded that maximum effectiveness can be achieved with green intervals of about 40 seconds (11).

Location in City -- The 1965 Highway Capacity Manual listed adjustments based on four area locations (2): central business district (CBD), fringe area, outlying business district, and residential area. Capacities (which are directly related to saturation flows) were highest in residential areas and lowest in the CBD.

An Australian report listed saturation flows for three environmental classes based on location of the intersection (6). The highest values were typically for residential areas, followed by industrial or shopping areas, and the lowest values were in the "city centre" area. Another Australian report used four categories very similar to those in the Highway Capacity Manual (5).

In a report from England, sites were classified as either good, average, or poor (1). Saturation flow was multiplied by 1.2 when the site was designated as good and 0.85 if poor. Interpolation between categories was done.

City Size -- The Highway Capacity Manual contained the only adjustment factor found for city size (2). Population of metropolitan area was combined with peak-hour factor to yield an adjustment factor. The area populations given ranged from 75,000 to one million.

An Australian study concluded there was no evidence of a relationship between saturation flow and city size (5). Slight adjustments for saturation flow were listed for several cities in which data were collected, but those adjustments were not related to city population.

A Canadian study indicated that saturation flows were dependent on the size of a community only when local traffic behavior was reflected (12). Local traffic behavior was concluded to be associated with other community characteristics that were not necessarily reflected in the population size.

Vehicle Type and Turning Maneuver -- Right-turning and left-turning vehicles have been observed to decrease saturation flow. Factors have been developed to convert turning vehicles into an equivalent number of through vehicles. Those adjustments varied by type of vehicle. In many instances, both turning maneuver and vehicle type were used to determine the number of "through car units". Therefore, those factors were considered together in this report.

Australian studies found that a left-turning car (the same as a right-turn in the United States) was equivalent to 1.25 through cars (5, 6). The through-car equivalent for a turning commercial vehicle was 2.5. A commercial or heavy vehicle was defined as any vehicle having more than two axles or having dual tires on the rear axle. One through commercial vehicle was found to be equivalent to two through cars. For unopposed right or left turns having a large radius of curvature, a through-car equivalent of 1.0 was used (6).

Unopposed left turns were converted to passenger-car equivalents (PCE's) in another report (13). The PCE for unopposed left turns was 1.2 for left turns made from left-through lanes and 1.05 from left-turn-only lanes. For opposed left turns, the PCE increased to 6.0 for an opposing volume of over 1,000 vehicles per hour. The passenger-car equivalents for right turns varied from 1.0 for light pedestrian activity to 2.0 for extremely heavy pedestrian activity. The recommended PCE value for trucks and through buses was 2.0. A truck was defined as a vehicle having six or more tires. A PCE value of 5.0 was recommended for each local bus (a local bus was one that made a scheduled stop at the intersection).

In another report, the average equivalents recommended for opposed left turns, using signal phases and opposing flow as input, were 2.9 for cars and 3.9 for trucks (14).

For unopposed right-turning vehicles (left turn in the U.S.) with exclusive lanes, Webster observed saturation flow to be related to the radius of curvature (r) measured in feet (1). For single lanes, saturation flow was given as 1,800/(1 + 5(r)). For dual lanes, total saturation flow was given as 3,000/(1 + 5(r)). Webster converted several vehicle types to passenger car unit (pcu) equivalents. The pcu equivalent was 1.75 for a heavy vehicle, 2.25 for a bus, 1.0 for a light-goods vehicle, 0.33 for a motorcycle, and 0.20 for a bicycle. For unopposed right turns (left in the U.S.) with no separate lane, these pcu equivalent values were to be used. The average opposed right-turning (left in the U.S.) vehicle was equivalent to 1.75 through vehicles. For left-turners (right in the U.S.), the guideline was to use a pcu value of 1.25 when left turns comprised over 10 percent of the traffic flow but no adjustment when less than 10 percent.

The Swedish Capacity Manual set base values for saturation flow at 1,700 vphg for lanes having only through traffic and 1,500 vphg for lanes having only turning traffic without conflict (15).

Another study converted turning movements and vehicle types to through car units (tcu's) (10). A right-turning passenger car had a tcu value of 1.20 while a right-turning truck had a tcu value of 1.81. A through truck had a tcu value of 1.63.

Passenger car equivalent (pce) values of 1.0 for right turns having a right-turn bay and 1.33 with no bay have been recommended (16). Other recommended pce values were 1.7 for through buses and 2.0 for left-turning buses.

Passenger car unit (pcu) values for several vehicle types were developed in a Canadian study (12). The pcu was 1.0 for a passenger car or small truck, 1.4 for a medium truck or bus, 2.4 for a large truck or semi-trailer, 0.6 for a motorcycle, and 3.5 for a special vehicle.

<u>Gradient</u> -- There have been varied recommendations concerning the effect of gradient on saturation flow. Several studies listed no factor for this variable. An Australian study recommended multiplying saturation flow by a factor equal to 1 + Gr/100 (6). Gr is the percent gradient, with a positive value for downhill grades and a negative value for uphill grades. This results in a one percent adjustment for each one percent change in grade. An earlier Australian report stated the effect of grade was to reduce saturation flow by about 1/2 percent for each one percent of uphill gradient (5).

A much greater effect of gradient on saturation flow was observed in a British study (17). That study noted that each one percent change in gradient caused a three percent change in saturation flow. Uphill grade decreased saturation flow and downhill grade increased it.

A Canadian study did not note any significant effects of uphill grades up to positive nine percent during summer traffic having homogeneous traffic flow and very few trucks (11). However, in winter, the effect of grade was significant. Adjustment factors were listed for uphill grades starting at three percent where trucks were present.

Lane Width -- Various studies have related lane width to saturation flow. In one study, the conclusion was that, for lanes 10 or more feet in width, lane width had little influence on flow rates (18). For lane widths between 9 and 10 feet, a reduction of 10 percent was recommended. Lane width did not include pavement used for parking.

One of the more detailed sets of adjustment factors was developed in an Australian report (6). A table listing lane width adjustment factors was presented. The factors were as follows:

Lane Width (Feet)	Adjustment Factor
8	0.88
9	0.93
10-12	1.00
13	1.03
14	1.045
15	1.06

The width at the narrowest point within 100 feet of the stopline was used. A reduction in saturation flow of 12 percent was shown for a lane width of 8 feet, and an increase of 6 percent was given for a lane width of 15 feet.

Adjustment factors for lane width were also given for use in the "critical movement analysis" procedure (13). These factors are applied to increase the effective passenger car volume, rather than to decrease the capacity. The following adjustment factors are used:

Lane Width (Feet)	Adjustment Factor
8.0- 9.9	1.10
10.0-12.9	1.00
13.0-15.9	0.90

Another Australian report generally concluded that lane width had little effect on saturation flow for lanes at least 10 feet wide (5). Adjustment factors from that report follow:

Lane Width (Feet)	Adjustment Factor
8	-12%
9	-7%
10	-1.5%
11	0
12	+1.5%
13	+3%
14	+4.5%

Some studies have concluded lane width did not have a significant effect on saturation flow. One study concluded lane width had a very small effect over the range of 6.5 to 16 feet (14).

Webster related saturation flow to approach widths rather than lane widths (1). For approaches between 10 and 13 feet wide, which would be one lane, saturation flows increased from 1,850 at 10 feet to 1,950 at 13 feet, a five percent increase.

A saturation flow manual developed in Canada contained a figure that related an adjustment factor to lane width (11). No adjustment factor was included for lane widths between 10 and 16.5 feet. A significant reduction in saturation flow was given for lane widths below 10 feet. For a lane width of 9 feet, about a 10 percent reduction was shown. For a width of 8 feet, an approximate 30 percent reduction was listed.

Turning Radius -- An Australian report give a separate level for saturation flow for a turning lane having a small turning radius (6). As previously mentioned, Webster listed equations relating saturation flow to radius of curvature for exclusive right-turning (left in the U.S.) lanes with no opposing flow (1).

Pedestrian Activity -- The effect of pedestrians is indirectly accounted for in the adjustment factor for location in city. The lowest values of saturation flow will be for locations in the CBD, where pedestrian activity is heaviest. Interruption of traffic flow causes lost time in the green phase, which reduces capacity.

Parking -- Webster observed that the reduction in saturation flow caused by a parked car near to the stop bar was equivalent to a loss in approach width at the stop bar (1). The effective loss in approach width was computed by the following formula:

$$\text{Loss in Width (ft)} = 5.5 - 0.9(Z-25)/K$$

in which Z = clear distance, in feet, from stop bar to parked car, and

K = green time, in seconds.

An Australian study investigated the effect of parked vehicles by parking vehicles at various distances back from the stop bar (5). The presence of parked vehicles was observed to decrease intersection capacity.

The Highway Capacity Manual gave separate adjustment factors to use, depending on whether or not parking was present (2). Parking was considered to be present when it was allowed within 250 feet of the intersection.

Use of a parking adjustment factor was included in a Canadian study (11). The factor depended upon whether the parking was upstream or downstream from the stop bar and upon lane width.

Approach Width -- Curves relating approach width to capacity were listed in the Highway Capacity Manual (2). One study evaluated use of approach width in the Highway Capacity Manual and recommended use of lane width instead of approach width (9). The procedure developed by Webster also related approach width to saturation flow (1). Saturation flow values for the various approach widths were as follows:

Approach Width (Feet)	Saturation Flow (Passenger Car Units per Hour)
10	1,850
11	1,875
12	1,900
13	1,950
14	2,075
15	2,250
16	2,475
17	2,700

The Swedish Capacity Manual related saturation flow to approach width and number of marked lanes (15).

Type of Lane -- Factors developed for turning maneuvers would include this consideration to a degree. However, it is possible to consider different types of through, right-turn, and left-turn lanes. An Australian study noted differences between types of these three basic groups of lanes to be statistically insignificant (5).

The capacity of dual left-turn lanes is a special case. The Highway Capacity Manual stated that, where two or more turning lanes were provided to handle a particular movement, the capacity of each additional lane was 0.8 times that of the first lane (2). One study noted the capacity per lane per hour of green to be 1,500 for double left-turn lanes compared to 1,700 for a single left-turn lane (9).

Time of Day (Peak or Non-Peak Hours) -- Saturation flow levels have been observed to be higher during peak periods compared to off-peak periods (16).

Speed Limit -- Adjustment factors specifically for various speed limits or operating speeds have not been used. This factor is considered indirectly through consideration of location in the city.

Weather or Road Surface Conditions -- Results relating capacity to weather or road surface conditions have varied. One study observed the effect of weather to be insignificant (5). Another paper stated that rain reduced capacities by eight to 24 percent (19). A Canadian study found headways to be higher during winter conditions when compared to summer conditions (12). Winter conditions were represented by snow on the pavement.

Light Conditions -- A comparison of saturation flows during daylight and night (street lights) conditions was made in one study (12). Results did not indicate any difference. Another study indicated saturation flow levels during daylight periods were slightly higher than levels during darkness (16).

APPENDIX B

CASE STUDIES

To illustrate the use of the saturation flow prediction formula given in the recommendations, a few case studies were conducted to compare measured and predicted values. The necessary approach data as shown in Figure 2 was collected at four lanes at different intersections in Lexington and Nicholasville. Saturation flow data were collected, and the measured values were compared to the value obtained with the saturation flow prediction formula. Following is a discussion of these calculations:

Case Study Number 1

Location	-- New Circle at Palumbo, Lexington		
Location in City	-- Outlying Business District		
Pedestrian Activity	-- Light		
City Population	-- 204,000		
Vehicle Distribution	-- Through Passenger Cars	--	97.4%
	Through Single-Unit Trucks	--	1.5%
	Through Combination Trucks	--	0.4%
	Through Motorcycles	--	0.7%
			100.0%

Gradient	-- 0 Percent
Lane Width	-- 12.7 Feet
Turning Radius	-- DNA
Type of Lane	-- Left of Two Through Lanes
Speed Limit	-- 45 MPH
Light Condition	-- Day

Predicted Value

S = 1650(Flp)(Fc)(Fvt)(Fg)(Fw)(Fr)(Ft)(Fs)(Fd)

Flp = 1.00
Fc = 1.00
Fvt = 100/((97.4)(1.00) + (1.5)(1.36) + (0.4)(2.02) + (0.7)(0.85) = 0.99
Fg = 1.00
Fw = 1.00
Fr = DNA
Ft = 1.00
Fs = 1.00
Fd = 1.00

S = 1650(0.99) = 1,634 vphg

Measured Value = 1,714 vphg
Difference = 4.7 percent

Case Study Number 2

Location	--	Nicholasville Road at New Circle, Lexington
Location in City	--	Outlying Business District
Pedestrian Activity	--	Light
City Population	--	204,000
Vehicle Distribution	--	Left-Turning Passenger Cars -- 88.9%
		Left-Turning Single-Unit Trucks -- 5.2%
		Left-Turning Combination Trucks -- 4.0%
		Left-Turning Motorcycles -- 1.9%
		100.0%

Gradient	--	+0.5 Percent
Lane Width	--	16.5 Feet
Turning Radius	--	DNA
Type of Lane	--	Single Left-Turn Lane
Speed Limit	--	45 MPH
Light Conditions	--	Day

Predicted Value

Flp = 1.00
Fc = 1.00
Fvt = 100/((88.9(0.98) + (5.2)(1.57) +
(4.0)(2.41) + (1.9)(0.85) = 0.94
Fg = 1-(0.5)(0.5)/100 = 1.00
Fw = 1.00
Fr = DNA
Ft = 1.02
Fs = 1.00
Fd = 1.00

S = 1650(.94)(1.02) = 1,582 vphg

Measured Value = 1,586 vphg
Difference = 0.3 percent

Case Study Number 3

Location	-- Main at Chestnut, Nicholasville		
Location in City	-- Central Business District		
Pedestrian Activity	-- Moderate		
City Population	-- 9,800		
Vehicle Distribution	-- Through Passenger Cars	--	89.8%
	Right-Turning Passenger Cars	--	4.8%
	Through Single-Unit Trucks	--	2.7%
	Through Combination Trucks	--	1.6%
	Through Motorcycles	--	1.1%
			100.0%

Gradient	-- +1.0 Percent
Lane Width	-- 12.5 Feet
Turning Radius	-- 16 Feet
Type of Lane	-- Through or Right-Turn Lane (Only Through Lane on Approach)
Speed Limit	-- 35 MPH
Light Conditions	-- Day

Predicted Value

Flp = 0.97
Fc = 0.87
Fvt = 100/((89.8)(1.00) + (4.8)(1.12) + (2.7)(1.36) + (1.6)(2.02) + (1.1)(0.85)) = 0.97
Fg = 1 - (0.5)(1)/100 = 1.00
Fw = 1.00
Fr = 95.2/100 + (.93)(4.8)/100 = 1.00
Ft = 0.96
Fs = 1.00
Fd = 1.00

S = 1,650(.97((.87)(.97)(.96) = 1,297 vphg

Measured Value = 1,309 vphg
Difference = 0.9 percent

Case Study Number 4

Location	-- Main at Oak, Nicholasville		
Location in City	-- Central Business District		
Pedestrian Activity	-- Moderate		
City Population	-- 9,800		
Vehicle Distribution	--	Through Passenger Cars	-- 79.1%
		Right-Turning Passenger Cars	-- 10.1%
		Through Single-Unit Trucks	-- 3.0%
		Right-Turning Single-Unit Trucks	-- 0.6%
		Through Combination Trucks	-- 6.0%
		Through Motorcycles	-- 1.2%
			100.0%

Gradient	-- -0.5 Percent
Lane Width	-- 14.0 Feet
Turning Radius	-- 13 Feet
Type of Lane	-- Through or Right-Turn Lane (Only Through Lane on Approach)
Speed Limit	-- 35 MPH
Light Conditions	-- Day

Predicted Value

Flp = 0.97
Fc = 0.87
Fvt = 100/((79.1)(1.00) + (10.1)(1.12) + (3.0)(1.36) + (0.6)(1.71) + (2.02)(6.0) + (1.2)(.85)) = 0.92
Fg = 1 - (1.1)(0.5)/100 = 0.99
Fw = 1.00
Fr = 89.3/100 + (.93)(10.7)/100 = 0.99
Ft = 0.96
Fs = 1.00
Fd = 1.00

S = 1,650(.97)(.87)(.92)(.99)(.99)(.96) = 1,205 vphg

Measured Value = 1,188
Difference = 1.4 percent

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